Large scale driving of concrete piles in stiff to very stiff clay

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ABSTRACT
BAMA Gruppen AS have constructed a new 44 000 m^2 warehouse, office and distribution centre in Groruddalen in northern parts of Oslo, Norway. Norwegian Geotechnical Institute (NGI) were employed as geotechnical consultants, contributing to evaluation of different foundation methods, design of pile foundations in cooperation with structural engineers and installation of piles in cooperation with the piling contractor. Within the perimeter of the warehouse, the depth to bedrock is between 30 to 50 meters. The typical soil profile is about 2 m of various fill material over 5 to 10 meters of stiff to very stiff clay over a slightly overconsolidated quick clay. Because of the extent of piling, concrete piles were very cost effective for the client. The stiff clay layer however could in theory lead to damaging tension stresses in the concrete piles during driving. For concrete piles up to 50 m length, the total number of blows is also considerable.

NGI performed pile-driving simulations, and the contractor performed test-driving at the site. During test-driving, PDA equipment was mounted, monitoring pile stresses during driving of the whole pile length. Test-driving was successful, and pile stresses observed within acceptable limits. During the early phases of production driving, the number of piles broken during driving was higher than expected. The large number of piles did however give good statistical data to interpret combinations of pile cross section and hammer weight and energy (fall height) which were unfavourable. In cooperation with the piling contractor, NGI continuously evaluated the piling production procedure and stop criteria. The aim was to reduce the number of broken piles, and to establish documentation of the bearing capacity for all piles. In total, the contractor drove about 1600 functional concrete piles, with a combined length of about 71 km.

Keywords: Concrete pile, steel core pile, stiff clay, quick clay

1 INTRODUCTION

Bama Gruppen AS is the largest fruit and vegetable grocer in Norway, distributing domestically produced and imported produce all over the country. In 2012-2013, the company constructed a new main warehouse, distribution centre and main office near Alnabru in the Groruddalen area in the north-eastern part of Oslo. The new warehouse has a base area of about 33 000 m^2 with about 44 000 m^2 of warehouse, distribution centre and office areas. Figure 1 shows the construction site location in Oslo, and *Figure 2* shows an aerial photo of the building more or less finished.

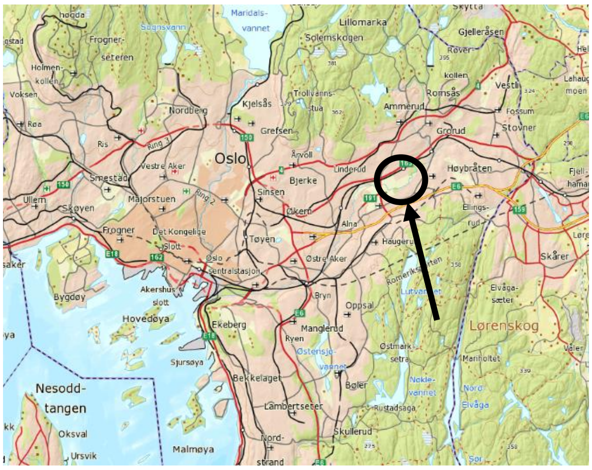

Figure 1: Construction location in Oslo

Figure 2: The finished building (source: bygg.no)

Norwegian Geotechnical Institute (NGI) had a history of studies and investigations on this property, both for the previous owner (ROM Eiendom) and Bama Gruppen. With the background history and knowledge on the site, NGI was hired as geotechnical consultants for foundations of the new building and a new bridge crossing a railway cut, as well as local slope stability improvements where needed.

2 THE CONSTRUCTION SITE

2.1 Brief history

The soil in Groruddalen is mainly composed of thick marine clay deposits. About 8300 years ago, a quick clay slide with a volume of 30 – 40 million m^3 covered the valley bottom (Eggestad, 1978). A moraine formation (the Alfaset morain) obstructed the slide masses from flowing freely further down the valley.

An overview of the slide area and the morain is shown in Figure 3. The slide masses are therefor still covering the "undisturbed" sediments in large areas northeast of the moraine formation, shown in Figure 4.

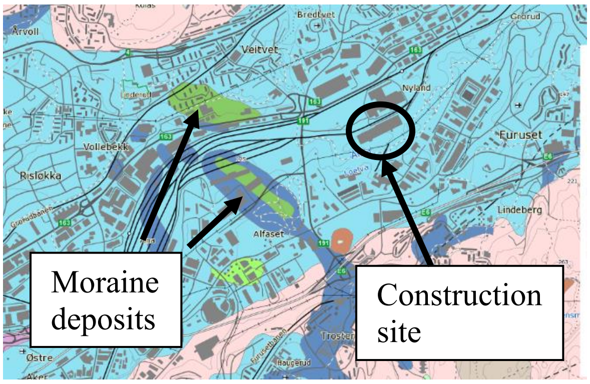

Figure 4: Geological map of the Alnabru area. Green is moraine deposits, blue is marine deposits and dark blue is beach deposits. (source: ngu.no).

In modern times, various landowners have used the construction site property as farming area, landfill, parking and storage area, sports area and most recently a rock crushing plant.

2.2 Topography

The construction site is situated on a plateau at about 118 to 120 masl, and is surrounded by ravines and cuts. To the east, south and southwest, the river Alna flows at the bottom of a ravine about 8 to 12 m deep. To the north, a railway freight line runs past the site. The railway line lies partially at the bottom of an open cut up to 10 m deep, and partially in an open concrete culvert. Figure 5 shows the warehouse location between the river ravine and railway cut.

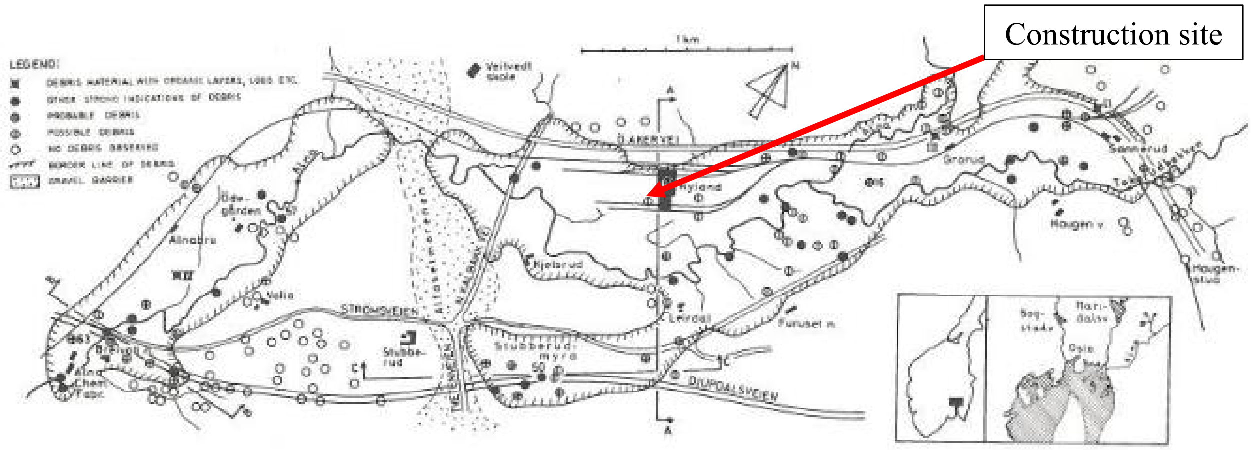

Figure 3: Overview of the Groruddalen quick clay slide (Eggestad, 1978)

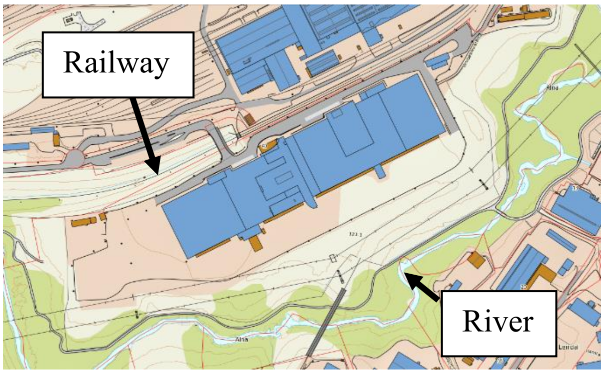

Figure 5: New warehouse in between railway cut and river ravine

2.3 Soil conditions

As previous owners had performed various studies for use of the property, several reports on soil investigations were already available. The evaluation of foundation methods and final design required detailed information, and a supplementary investigation programme was performed by NGI. The supplementary investigation programme included total soundings, rotary soundings, cone penetration testing, soil sampling and measurement of in situ pore pressures by hydraulic piezometers.

Both auger samples of the top fill layer and undisturbed samples of the lower clay layers were analysed in the laboratory for soil classification, routine parameters and oedometer tests.

The general soil profile at the site is about 1-2 m of various fill material over a stiff to very stiff clay layer. The stiff clay layer is the reconsolidated landslide masses from the quick clay slide mentioned earlier, and has a thickness of about 5 to 10 meters at the construction site. Below the landslide masses, the "undisturbed" clay is quick and slightly overconsolidated. Figure 6 shows a general profile of undrained active shear strength based on interpretation of CPTU soundings. The red line is the shear strength of a normally consolidated clay for comparison.

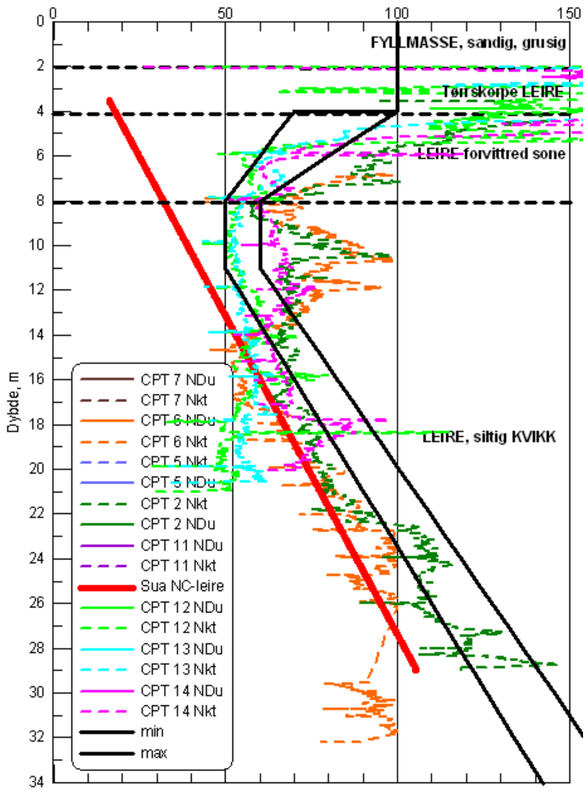

Figure 6: Undrained active shear strength based on CPTU soundings

In southwestern parts of the site the quick clay continues down to bedrock, while in the northeastern parts a moraine layer up to about 15 meters thick covers the bedrock. The depth from terrain to bedrock varies between 30 to 50 meters.

The ground water level was measured to about 3 m below terrain, with a slightly lower than hydrostatic distribution with depth caused by drainage to the ravines.

3 PLANNING

3.1 Evaluation of foundation methods

NGI performed evaluation of various principles for foundations for the new warehouse. Different variations of direct foundations on terrain and pile foundation to bedrock were considered.

The scenarios considered for direct foundations were different levels of adding and/or removing fill material at the site to establish the finished floor at different height levels. Partly removing existing soil top layer and replacing it with lightweight fill material was also considered. Settlement calculations showed that long term settlements without the use of lightweight fill materials were in the order of 20 – 25 cm. Replacing the top layer with lightweight fill material would give a slight reduction in settlements of about 5 – 10 cm.

Settlements in the order calculated were considered to the acceptable for the warehouse structure, but differential settlements would cause cracking and increased repair and maintenance of the floor. The users of fresh produce warehouse could not accept this uncertainty regarding structural maintenance. Pile foundations were therefor concluded to be the only alternative.

3.2 Pile foundations

Only end-bearing piles to rock or sufficient depth in moraine were considered, as friction-bearing piles would be difficult in the low plastic sensitive clay. The piles types considered were concrete piles, massive steel piles (H-profiles) and bored steel core piles.

The freely supported deck of the warehouse and the structure itself required a certain number of piles almost regardless of the capacity of each pile. Concrete piles would therefore be significantly cost effective compared with the steel piles. The logistics of the large number of piles that were to be installed during a short period also favoured the concrete piles. The planned amount of concrete piles for this project was about equivalent to the total volume of concrete piles casted in Norway in a poor production year.

An applied layer of bitumen coating reduces the effects of negative skin friction on the bearing capacity of all piles.

Pile design in general was done in accordance to the Norwegian pile design handbook Peleveiledningen (The Norwegian Pile Committee, 2005).

4 VERIFICATION OF USABILITY

4.1 Pile stresses during driving

The soil characteristics at the construction site does not favour the concrete pile. The penetration from the stiff to very stiff clay down into the softer quick clay represents a risk of significant tensions stresses in the piles during driving. Most of the piles would also be very long, increasing the risk of breakage as the numberof blows on the piles would be considerable.

A test-driving programme was established at the actual construction site, to test the performance of the piles in the actual soil conditions. NGI performed GRLWEAP analysis of the driving process, concluding that the margin regarding tension stresses should be within acceptable range.

4.2 Test-driving

The test-driving was performed in late June and early July of 2012. A total number of 24 P270NA piles and 2 P230NA were driven using a 60 kN hydraulic hammer. The piles were scattered over the construction site, covering different variations in ground conditions. Out of the 26 test-piles, 2 piles broke during driving. The setup of test-piling at the site is shown in Figure 7.

During test-driving, 7 piles were driven with PDA-sensors mounted. The sensors logged the whole driving process of the piles, from terrain and down to stop criteria when hitting either bedrock or moraine layers. This gave an output of the tension stresses in the piles for all blows during the driving history. Results from PDA measurements confirmed previous analysis results that tension stresses were within acceptable range for the piles, and the project could continue with concrete piles as planned.

Figure 7: Test piling with PDA-testing

5 VERIFICATION OF BEARING CAPACITY

The piles would reach a bearing tip resistance at either rock or in frictional soil material (moraine). Two different procedures stop criteria was developed, and the crane operator would have to use the appropriate procedure based on if he believed the pile had reached rock or moraine.

Stop criteria procedures are normally based on gradually increasing hammer energy, ending up with a number of blows with high energy. Because the piles had already suffered a very high number of blows during driving, the energy during stop driving was reduced to half of what was evaluated to be a normal procedure. Instead, one single control blow with high energy was performed for all piles at the end of the stop criteria driving. Elastic compression and final set was measured for all piles during the control blow, and this information was used to evaluate if the pile had reached the design bearing capacity.

All piles were restruck after a few days, and generally did set a few millimetres during restrike. Another high energy control blow with measurement of elastic compression and set was performed at the end of restrike for all piles.

6 PRODUCTION PILING

6.1 Effect of hammer size

Production piling started in the end of August 2012. The piling contractor mobilized four different piling rigs, with 60 kN, 70 kN, 90 kN and 100 kN hydraulic hammers. The two lighter ones were used for P230NA and P270NA piles, while the two heavier ones were used only for P345MA piles.

The different hammer sizes did, as could be expected, perform differently in terms of percentage of broken piles. Table 1 shows the total number of piles driven with each hammer, including broken piles.

Table 1: Total number of piles driven by hammer size

Hammer	P230NA	P270NA	P345MA
60 kN	398	385	-
70 kN	90	645	-
90 kN	-	-	130
100 kN	-	-	110

Figure 8 shows the distribution of broken piles by cross section and hammer size. The most obvious result is the combination of P230NA piles and 70 kN, which proved to be significantly unfavourable. Driving of the smallest piles with this hammer was therefore stopped completely as soon as the broken piles percentage was identified to be that high compared with other combinations. In general, P230NA and P345MA piles proved to be more difficult with respect to pile breakage compared with P270NA piles.

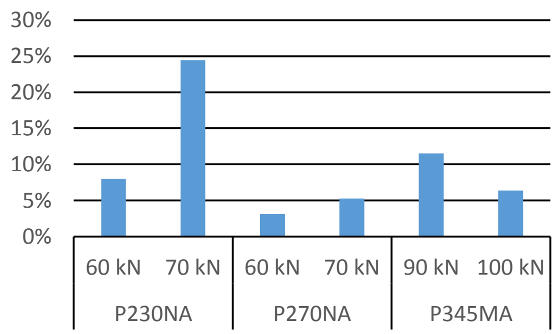

Figure 8: Percentage of broken piles by cross section and hammer size

Another observation from Figure 8 is the fact that the combination of P270NA piles and 60 kN hammer weight was the best combination with respect to number of broken piles. This was the same combination that was used during test-driving, and one could therefor say that the test-driving setup was not able to capture some difficult aspects of the pile-driving job at the site.

6.2 Effect of driving energy

The energy used during pile driving was evaluated from the pile driving analysis and the test-driving. After a large enough number of piles was driven to give statistical indications, driving energy was reduced as much as possible for P230NA and P270NA piles. The contractor still has to have some energy for him to be able to have a reasonable production. For the larger P345MA piles, further energy reduction was ruled out as production speed was already quite low.

Table 2 gives an overview of the total number of driven piles distributed by cross section, hammer size and driving energy. The driving energy is expressed as the free fall height of the hammer.

Table 2: Number of driven piles by cross section, hammer size and driving energy

Energy	P230NA		P270NA	
	60 kN	70 kN	60 kN	70 kN
10 cm	4	1	3	
15 cm	80			
20 cm	285	56	158	532
25 cm			20	
30 cm	28	33	182	104
35 cm				2
40 cm			17	

Energy	P345MA	
	90 kN	100kN
20 cm	11	12
25 cm		
30 cm	13	52
35 cm		43
40 cm	105	3

Figure 9 illustrates the percentage of broken piles by driving energy. Combinations with fewer than 10 piles (ref. Table 2) are left out of the diagram. The hammer-pile-combinations P230NA-70 kN and P345MA-90 kN obviously stand out as generally unfavourable. For other combinations, reducing the energy during driving clearly reduces the number of broken piles. Reducing the energy increases the number of blows required to get the pile down to bearing layers, increasing the sum of dynamic loading on the pile during installation. From reduced energy also follows reduced stresses in the pile during driving, and the reduced stresses means less chance of the pile breaking over time.

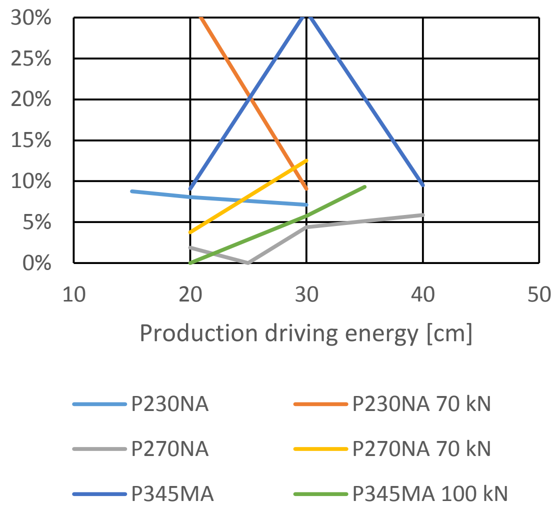

Figure 9: Percentage of broken piles by cross section, hammer size and driving energy

The effect was especially significant when reducing energy from 30 to 20 cm for P270NA piles driven with 70 kN hammer. This combination was used for about 1/3 of the total number of piles in the project (Table 1).

6.3 Effect of pile length

The columns in Figure 10 show an overview of the total number of piles for each cross section and total pile length interval. The graph lines illustrate the percentage of broken piles for the corresponding cross section and total pile length interval.

According to figure 9, the number of broken piles increases significantly when the total pile length increases over about 46-50 m. Almost all P230NA and P345MA piles and about 30 % of P270NA piles longer than 50 m were broken. About 15 to 20 % of P230NA and P345MA piles and 7 % of P270NA piles between 46 to 50 m long were broken.

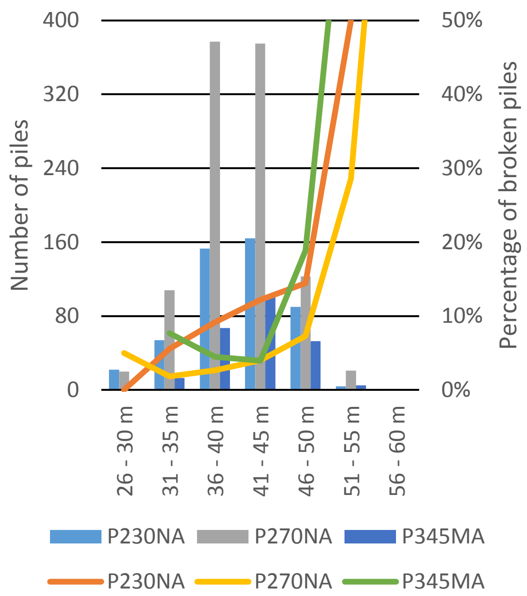

Figure 10: Number of piles and broken piles by pile length

An increasing pile length increases the risk of some kind of unfavourable curvature of the pile, increasing the risk of breakage. The increased length also comes with an increase in total number of blows and cycles of dynamic stresses.

6.4 Steel core piles

Broken concrete piles, or piles which were suspected to be broken, were replaced by drilled steel core piles. Due to earthquake design, some piles for the warehouse structure needed tension capacity, and the steel core piles were therefor always a part of the plan. The logistics and equipment was available at the site, and pile replacement could be designed immediately as the contractor reported broken piles.

The soil conditions at the site also proved difficult during drilling of casings for the steel core piles. During drilling the drill bit would get stuck, sometimes breaking the casing itself causing loss of drilling equipment.

7 OTHER INCIDENTS

7.1 Shallow slope failure

In September 2012, after a heavy rainfall, surface cracks were observed in the slope towards the railway line. The appearance of the crack coincided with pile driving near the top of the slope. Figure 11 shows a picture taken from the other side of the railway cut, with the pile driving cranes working in the background.

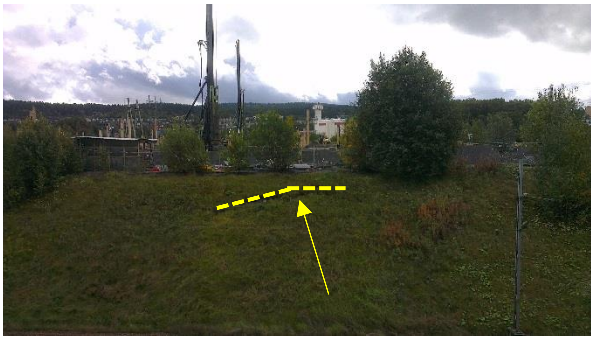

Figure 11: Surface crack in slope (marked with yellow dashed line)

Even shallow surface slides could cause concerns regarding operation of the railway line, and driving of concrete piles near the slope was stopped immediately. The slope was instrumented with electronic piezometers and inclinometer casing. The piezometers measured a very high pore pressure, with a magnitude in the order of the same as the vertical overburden (meaning that the effective stresses were very small). Inclinometer measurements were done with short time intervals for the first period of time after installation, but indicated no significant movement in the slope.

For the slope to have a sufficient factor of safety in a permanent situation, the top couple of meters of the slope was planned to be removed and replaced by lightweight fill material. This operation was expedited to increase the safety during the construction phase. Drilled steel core piles replaced the concrete piles within a safety zone from the slope, in order to prevent further build-up of horizontal stresses and pore pressures. The drilled steel core piles was installed as gently as possible (low air pressure and drill speed) to reduce the risk of further set up of excess pore pressure.

7.2 Thickness of bitumen coating

It was discovered during a late phase of pile installation that the bitumen cover on the piles appeared to the significantly thinner than specified. According to the specifications in NS 3420, the thickness on concrete piles should be at least 2 mm. Measurements later done by both factories which delivered piles to the site confirmed that the layer most probably was generally thinner than specified, often less than 1 mm. The client wondered if the reduced thickness had any implications for the pile capacity, which at this point in the construction process would have been fatal.

According to Claessen & Horvat (1974), shear stress transferred from the soil to the pile through the bitumen layer depends on the shear stiffness (or viscosity) and the thickness of the bitumen layer. The shear stiffness is depended on the temperature of the bitumen layer and the rate of strain. At a high rate of strain, when driving the pile, the layer is stiff and resists wear. At a slow rate of strain, when the soil settles around the pile, the layer is softer and does not transfer much shear stress to the pile. Furthermore, the shear stress transferred through the bitumen is inversely propotional to the thickness of the bitumen layer.

Studies were performed to establish reliable values of the negative skin friction affecting the piles, knowing the actual thickness of the bitumen layer. The increased strength of the piles, taking into account strength development of the concrete after the 28 day design strength, was also evaluated. In total, the structural and geotechnical engineers concluded that there was very little reason to doubt that the piles did have the capacity they were supposed to have, and mainly because the evaluation of negative friction on the piles fortunately had been on the conservative side during the design stage The discovery of the reduced bitumen thickness did raise a few questions in this project. Apparently, the pile factories followed their normal procedures, meaning that there probably are other projects where the bitumen thickness is thinner than specified in the Norwegian standards. Bitumen thickness should be measured after application at the factory and by the contractor at the construction site before the piles are used.

8 SUMMARY AND CONCLUSIONS

Foundations for the warehouse were successfully established with concrete piles supplemented with steel core piles. Despite the difficult ground conditions and the long pile lengths, the solution ended up being effective especially considering costs for the client. The percentage of broken piles was high, but counter-measurements such as close monitoring of unfavourable conditions, dialogue with the contractor and adjustment of the piling procedure when needed was important factors in bringing the number as low as possible. 1758 concrete piles with a combined length of around 71 km were driven. The final ratio of broken piles was about 7 %.

Test-driving was performed with mainly one pile cross section and one hammer size. This combination later proved to be the most favourable one, and other combinations of pile cross section and hammer size were more difficult during the construction phase. Testing all relevant combinations of hammers and cross sections before driving had started could have resulted in better or faster tuning of driving procedures, but the test-driving would also be quite a bit more resource demanding.

Another aspect was the short period of time from test-driving to the start of the actual construction. If the test-driving had ended up proving that the concrete pile would be unsuitable, the whole project would probably have been delayed and costs would have escalated while waiting for new solutions to be implemented.

The effects on slope stability could have been investigated further earlier in the project, but in the end the solution with drilled piles near the slope would most likely have been used anyway.

Reducing negative skin friction on piles is depended on the thickness of the bitumen layer, not just the application of a layer of random thickness. If important, thicker layers could be specified. Layer thickness should be measured before piles are used.

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